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INCLUDING PRESTRESSED CONCRETE

APRIL 1956 VOL. LI. NO. 4

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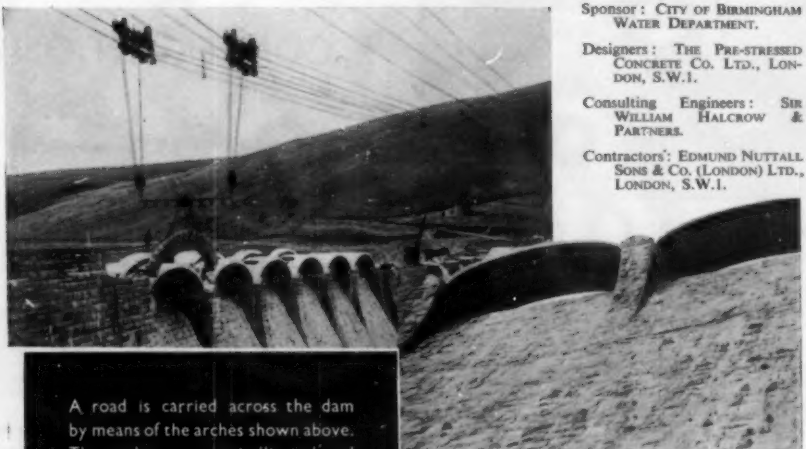

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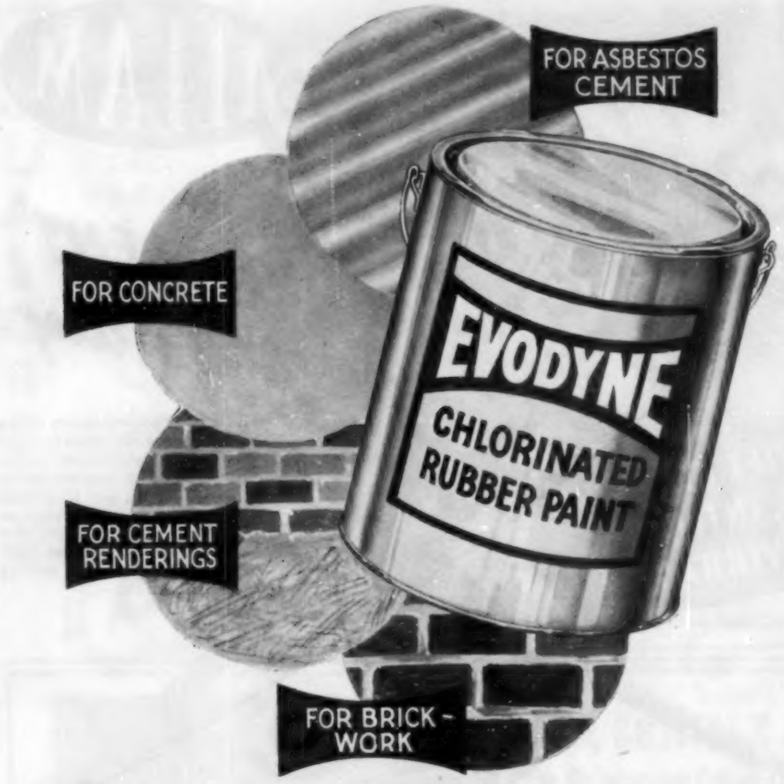

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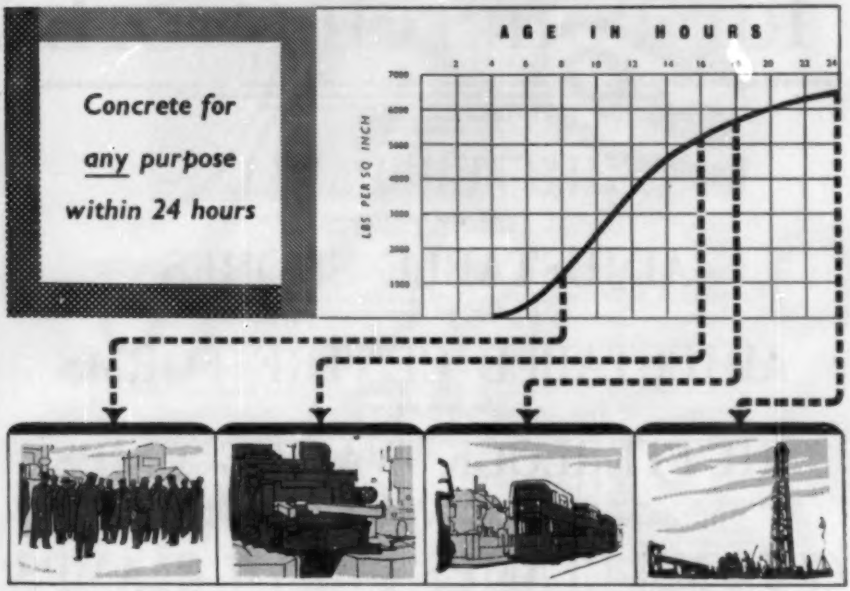

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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Volume LI, No. 4. LONDON, APRIL, 1956.

EDITORIAL NOTES

The Durability of Concrete.

To declare that something is impossible merely because we cannot conceive it happening, or to believe that assumptions made on evidence available will not be proved to be wrong in the future, are common human failings that are often summed up in the words " it is foolish to prophesy ". It is probable that the builders of the Pyramids and the mediaeval cathedrals believed that their works would stand for eternity, and they would not have thought it possible that so much crumbling stone would have to be replaced, or that so much repair and propping would have to be done, within a few hundred years. The Romans also may have thought that their concrete would endure for ever, but very little of it is to be seen to-day. In more recent times Sir Charles Barry and Augustus Pugin, the architects of the Houses of Parliament, could have had no idea that the whole of the stone facing would have to be renewed less than a hundred years after the buildings were completed. There is therefore every excuse for the belief of the pioneers of reinforced concrete that it would be more permanent than other building materials. They believed that, by crushing natural stone and reconstructing it with Portland cement, they were providing a building material that would be stronger and more durable than many natural stones composed of grains of rock cemented together by Nature, and the slogan Concrete for Permanence was in common use.

The passage of time has proved this to be wrong, or at any rate an exaggeration, even when it is remembered that nothing in Nature is permanent and the word was used in a relative sense. In less than ten years many of the early structures had extensively deteriorated. In the past twenty-five years the repair of early structures by the gunite process has become an important industry in many countries where concrete structures have been in existence for fifty years or even shorter periods. Some of the trouble has been due to the effects of frost entering and disrupting the concrete, but there is no doubt that most of it is due to bad design or bad workmanship in providing too little protection to the steel reinforcement. Whether the deterioration is due to porous concrete or to insufficient cover to the reinforcement, the structures including these defects are those that were recommended because of their assumed permanency and freedom from deterioration.

The lessons to be learned from deteriorated concrete structures are few and simple, but they are very important. The first is that if exposed concrete is to endure it must be resistant to the action of frost, and the other is that when it is reinforced the cover of concrete must be sufficiently thick and impermeable to prevent moisture from attacking and corroding the steel. It would seem that some of the knowledge of the pioneers was not properly appreciated by other engineers in this country when they started to design in reinforced concrete. For example, at Southampton a jetty designed and supervised by Hennebique in 1899 is still in use and to-day there appears to be no sign of deterioration, whereas a quay built nearby by the railway company in 1902 had so badly deteriorated that it had to be largely rebuilt a few years later. Some maritime structures built early in the century by Considère are also in better condition than similar structures designed by other engineers, probably because Considère specified a rich mixture (800 lb. of cement per cubic yard) and thereby provided better protection to the steel. For durable concrete it is important that the engineer should provide ample cover of concrete to the steel and that the contractor should ensure that this cover is in fact present in the completed work.

In our January number it was assumed by one author that the maximum useful life of a concrete structure built early this century is fifty years. The author of a paper referred to on page 351 of this number expresses the opinion that a reinforced concrete structure might have a useful life of a hundred years provided that it is resurfaced twice in that period, and he sees no reason to believe that concrete structures built to-day will be any more durable than those built fifty years ago. We would not attempt to counter this prophecy, but it should be mentioned that the knowledge gained in recent years tends to give a more optimistic view of the future of concrete structures now being built. For example, in the U.S.A. and elsewhere deterioration has been traced to the reaction with cement of alkali present in the aggregates, and this source of failure can be prevented by not using such aggregate. There is much better supervision of concrete construction nowadays and contractors have much more experience, with consequently less risk of bad workmanship. The phenomenon of the "creep" of concrete is known and can be allowed for in design. Modern plant enables more accurate proportioning of mixtures, and greater density is achieved by vibratory methods of consolidation or by vacuum processes. The importance of the water content is better understood, and as a result stiffer concretes are commonly used. It is also now recognised that compressive strength and density are not the only criteria of durability, and that the use of air-entraining agents producing concrete of less density and less compressive strength also results in greater resistance to frost and consequent greater durability. There is much evidence (but no proof) to suggest that concrete structures built to-day will have a much longer life than those built even twenty years ago provided that full advantage is taken of present knowledge and that the work is strictly supervised and conscientiously done. Experience shows that improvements in the quality of concrete may be valuable in producing more durable structures, and should not be lightly nullified by methods of design in which these improvements are fully taken into account in an effort to cheapen concrete construction. This might well be borne in mind by the advocates of the ultimate-load method of design when they claim that it permits important economies of materials to be made.

Prestressing with Stranded Wires.

THE LEONHARDT AND LOEBA SYSTEMS.

THE following account of the use in Germany of stranded prestressing wires is abstracted from a lecture by Dr. Fritz Leonhardt given in London last year to the Prestressed Concrete Development Group. The methods described have been used in more than 300 bridges and other structures.

DUCTS FOR CABLES.—The wires must be at least the length of the structure so that no splices will be necessary. Wires with seven strands of 0·12 in. diameter wires are preferred, because they are made in lengths of about 4000 ft. and can be handled easily. The wires are placed in horizontal layers in a rectangular steel sheet casing (*Fig.* 1) and horizontal spacers with vertical projections keep them apart to facilitate the flow of grout. Suitable spacings are 0·04 in. for stranded wires and 0·06 in. for smooth wires. These spaces should be uniform throughout, as tests in glass casings have shown that grout will fill all the voids only if the resistance to flow is almost equal across the whole cross section. The spacers are at least 3 ft. apart. For large cables with a cross section of, for example, 6 in. by 6 in. or more, the casings are of corrugated steel, and the joints are welded.

REDUCING FRICTION.—To reduce friction when the wires are tensioned, a polygonal cable-line is used with only few changes of direction between long straight portions. The places where the cables change direction are curved to a radius of between 16 ft. and 50 ft., resulting in curved portions 8 in. to 16 in. long. The short curved casings are welded together of sheet steel about $\frac{3}{16}$ in. thick to provide a stable curved plate for distributing the force evenly to the concrete. The width of the curved portion is equal to the width of the cable in order to avoid displacement of the wires. The straight portions are about $\frac{3}{8}$ in. wider and deeper than the curved portions so that there is theoretically no contact between the cable and the casing in these sections during prestressing, and therefore no friction. The only contact and friction are at the short curved portions, and here it can be decreased by lubrication. For this purpose, two hard and smooth steel sheets, with paraffin wax between them, are placed on the cable below the curved cover-plate. This method theoretically decreases the friction from 0·2 to 0·002. Similar plates can be used at the sides of the short curved casing if there are many changes of direction or if the cable is curved horizontally also. For a single span or a continuous beam of two spans side-plates are not necessary, because the sliding plates on the inside of the curve are sufficient to reduce the friction below 0·1. Thus, in this method friction is not important and grouting difficulties are overcome. The single large cable enables grout to be injected at the low points with outlets at the high points.

Fig. 1.—Cable in Sheet-steel Trough.

GROUTING THE CABLES.—The channel is filled with water before injecting grout because water is replaced much more readily than air and protects the grout at the head of the flow from losing moisture, as it would when injected against air. Only a short portion of the grout becomes mixed with the water, and this is pumped out at the outlet.

The water-cement ratio of the grout is about 0·42, and an expanding ingredient, such as aluminium powder, is added so that the grout will exert a slight pressure during hardening.

ADVANTAGE OF CONCENTRATED CABLES.—Tests have proved that the concentration of the prestressing steel in a few large cables is not a disadvantage if grout of sufficient strength is used and if the casings have corrugated or otherwise roughened walls to ensure a high bond strength. The bond strength is from 400 lb. to 500 lb. per square inch.

A large cable needs a smaller area than a number of small cables for the same force, because small cables must have such spacing as is necessary for sound and dense concrete to flow around each duct, whereas around one large cable stiffer concrete with coarser aggregate can be placed and well vibrated, resulting in a higher strength and less shrinkage and creep. This is shown in *Fig.* 2, which indicates the spacing of the cables to produce a prestressing force of 1000 tons by the author's method and by other methods. The axis of the cable must be very exact because, with such

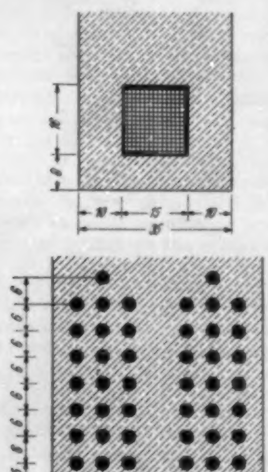


Fig. 2.—Comparison of Distribution of Cables.

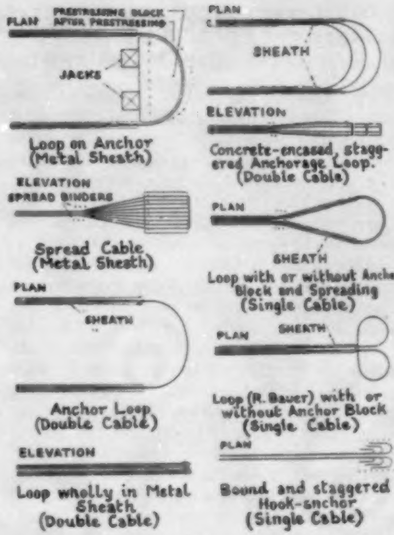


Fig. 3.—Types of Anchorage.

large forces, small displacements change the moments greatly, but a single casing can be placed, supported, and checked for its correct elevation much more simply and exactly than a number of small ducts. There is a marked economy in the use of one large cable due to the small amount of labour required for placing the steel, for jacking, and for grouting.

ANCHORAGE.—To anchor the cables at one end of the structure, a loop (*Fig.* 3) is usually fixed in the concrete. At the other end the cable is placed around a concrete block (*Fig.* 4) which is moved by hydraulic jacks acting against the hardened structure. Special loops, or vertical loops, are used if the main beams are far apart, or for single beams. *Fig.* 4 shows a movable prestressing block with the loop anchorage of many strands on its curved face. In one case as many as 372 strands in five layers were arranged on such a block, and tensioned with a force of 3700 tons.

For wide and shallow bridges several main beams with several looped anchors and stressing blocks are used. *Fig.* 5 shows four blocks, with the hydraulic jack connected to a high-pressure pump

Fig. 4.—Anchor Block and Cables.

Fig. 5.—Anchor Blocks and Jacks.

so that the whole prestressing operation can be done in from one to four hours depending on the length of the structure. After tensioning the cables, the gap between the blocks and the bridge is filled with precast blocks or with high-early-strength concrete. The jacks are then removed, the jack pockets filled, and the stressing blocks covered with concrete.

The extra concrete required for these loop anchorages is economical only in large structures. For smaller spans and for precast beams a bond anchorage is formed by fanning out the wires of the cable at the ends so that there is enough space to vibrate concrete around each wire. Oval wires with diagonal ribs are preferred because of their good bonding qualities. The same good anchorage by bond is also obtained if the ends of smooth wires are crimped for a length of 1 ft. or 2 ft. according to the diameter. The jacks are placed immediately behind this anchorage (*Fig.* 6) which is moved away a distance equal to the elongation of the cable. Cables tensioned with a force of up to 1000 tons can be anchored in this way. For continuous beams in Brazil the prestressing joint and the jacks were placed above the intermediate piers, and the complete end spans moved longitudinally with the stressing forces.

TENSIONING THE CABLES. — This method allows the cables to be tensioned in several stages. The first application of 10 to 20 per cent. of the total initial force is made one or two days after the concrete has been placed in order to prevent early cracks due to shrinkage. This slight compression during the early hardening period increases the tensile strength of the concrete and decreases the creep after the prestressing force is applied. The second tensioning is not done before the concrete is hard and the force then applied should be sufficient to carry the dead load without tension in the concrete. The centering is then lowered to make the dead load more effective, and the full initial prestressing force can then be applied. By this means long continuous structures have been built without cracks.

JACKS.—Large hydraulic jacks (*Fig.* 7) of 300 or 500 tons capacity have been designed with small friction of the piston. The hollow pistons fit to the cylinder only at the seal, and they have some "play" for the remainder of their length. Thus, the piston is self-adjusting to small changes in the alignment of the concrete surfaces in relation to the axis of the cable.

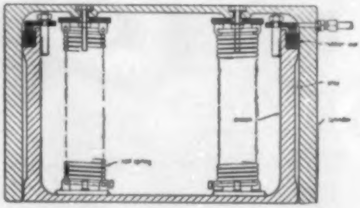


Fig. 7.—Hydraulic Jack used in the Baur-Leonhardt System.

A RAILWAY UNDER-BRIDGE.—The railway bridge built on a skew at Heilbronn has continuous beams of hollow section. It has five spans of 60 ft. to 70 ft. and a depth of only 3 ft. 6 in. It was built in 1950 to 1951. *Fig.* 8 shows the continuous cables and the prestressing blocks. The high points of the cables are over the piers. A width of 56 ft. was prestressed with seven prestressing blocks and a prestressing force of 8100

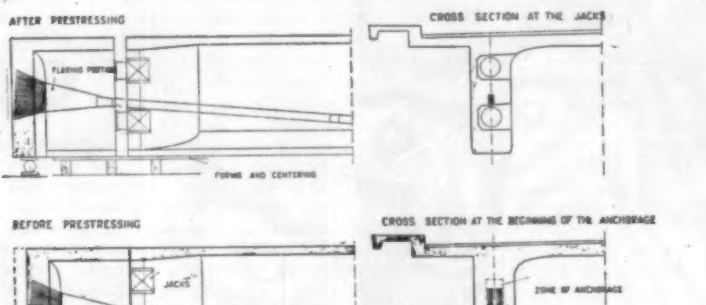


Fig. 6.—Bonded Anchorages.

Fig. 8.—Railway Bridge at Heilbronn during Construction.

tons. Each of the cables had 396 strands. Open construction joints were left above the piers to prevent cracks due to the settlement of the centering. These gaps were filled with concrete before the first stage of the prestressing was carried out. The transverse prestressing had already been done. There are no cracks in this wide and long structure and it has carried heavy main-line railway traffic since 1951.

THE LOEBA SYSTEM.—A bridge of great slenderness is the two-hinged frame across the river Neckar at Stuttgart-Cannstatt, with a span of 225 ft. and a depth of 4 ft. for a great part of its length (*Fig.* 9). The cross section (*Fig.* 10) shows that only two longitudinal box-girders are used, with prestressed transverse girders between them. The transverse prestressing was done with Loeba members composed of twelve wires of 0·2 in. diameter. In this system the wires are placed in a rectangular sheath in two horizontal layers with spacers between. At one end there is a loop around a small steel anchor-piece, held by a threaded bar for mounting on the shutters and later for prestressing. A conical shell and rubber piece form the space for the loop-anchor to move during prestressing. These parts are shown in *Fig.* 11. The other end of the cable has a bonded anchorage, with deformed wire ends within a spiral reinforcement. A steel plate is placed over the threaded bar and over the circular hole formed by the rubber piece. A nut holds the bar and plate in position. A small jack (*Fig.* 12) weighing only about 22 lb. is then attached and moves the looped anchor-piece. The elongation is retained by screwing the nut against the steel plate so that the jack can be released. The cable is then grouted through the steel plate. As soon as the grout has hardened the steel plate and the bar can be taken off, because the loop anchorage rests safely on the hardened grout which is spirally reinforced. No steel parts project from the concrete, and only a small circular hole has to be filled with concrete. More than 10,000 such cables have already been used.

AN UNUSUAL HINGE.—At one side of the river the bridge shown in *Fig.* 9 had to be founded on the piled foundation of a former bridge which was not strong enough to resist the horizontal force of the new bridge. The hinge of the bridge on this side was therefore made horizontally movable, and a slab was placed

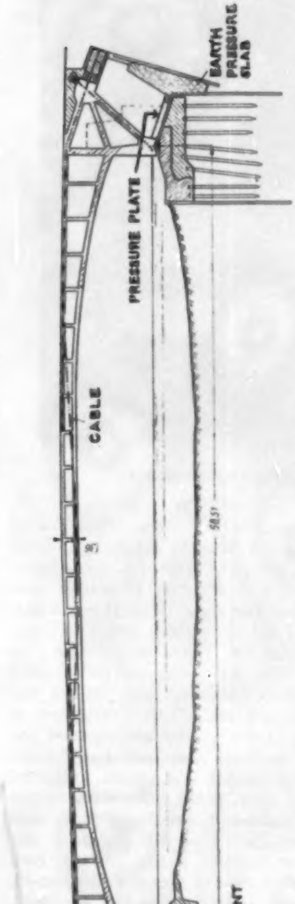


Fig. 9.—Bridge at Stuttgart.

Fig. 11.—Loop Anchorage (Loeba System).

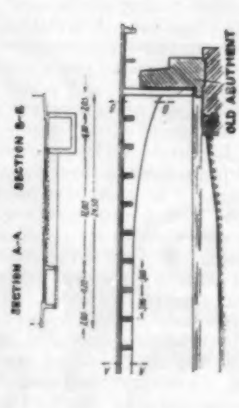


Fig. 10.—Bridge at Stuttgart.

Fig. 12.—Jack used in Loeba System.

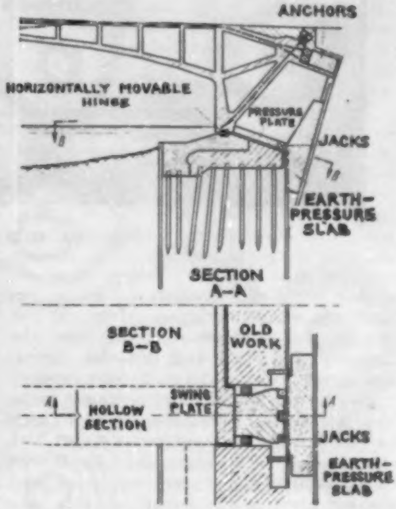


Fig. 13.—Abutment of Bridge at Stuttgart.

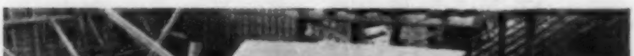


Fig. 14.—Anchor Blocks at Bridge at Stuttgart.

Fig. 15.—Bridge at Stuttgart.

almost vertically against the filling behind the old foundation (*Fig.* 13). A sloping slab was provided between the hinge of the frame and this slab, transferring the horizontal thrust to the ground.

By the use of suitable hydraulic jacks the horizontal resistance of the earth can be fixed to the required amount, and the anticipated movements of the bridge due to shrinkage and creep can be allowed for by moving the hinge on this side horizontally. Without this device, the deflection due to shrinkage and creep would have amounted to 11 in. after a few years. The deflection had to be adjusted after the first summer, and it is expected that it will have to be done again after the third or fourth summer,

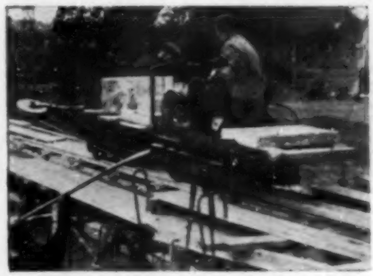


Fig. 16.—Placing Cables Mechanically.

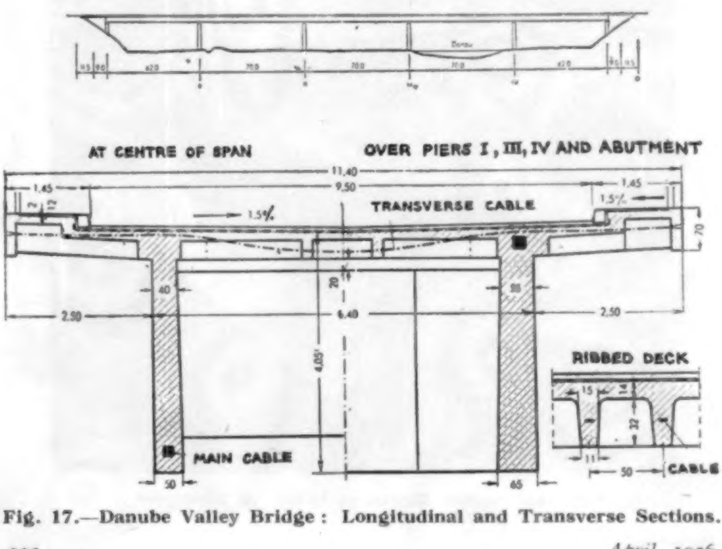


Fig. 17.—Danube Valley Bridge: Longitudinal and Transverse Sections.

when the creep of concrete will be negligible. *Fig.* 14 shows the large anchor-block for the main cable, and also five smaller anchor-blocks for the vertical cables in the pier. The legs of the frame are so stiff that there is no vibration in the bridge in spite of its great slenderness (*Fig.* 15). The concrete in the bottom slab of the hollow beams had a compressive strength of 8500 lb. per square inch, and the longitudinal prestressing force was 7200 tons.

CONTINUOUS BEAMS WITHOUT EXPANSION JOINTS.—Several bridges have been built with parallel continuous beams over 4, 5, 6, and 7 spans which have been prestressed in one operation through all the spans, thus avoiding expansion joints in bridges as long as 1200 ft.

PLACING THE CABLES.—The method of placing the cables in a bridge with five spans of 140 ft. each is shown in *Fig.* 16. A rail track was built slightly above the deck. A small Diesel engine pushed a small truck, carried on a horizontal beam, with two horizontal wheels just above the cable casings. A loop of the cable was placed around these wheels and the truck driven to the other end of the bridge, unwinding the cable from the reel behind the abutment. The cables dropped into the casing automatically, and when six or eight cables had been placed the spacers were placed in position. The placing of the four prestressing cables of this bridge, each 750 ft. long, was done in five days.

CONTINUOUS BRIDGE 1150 FT. LONG.—The largest prestressed continuous bridge so far is the bridge across the Danube near Untermarchtal with five spans of up to 230 ft. It was built in two parts, of two and three spans respectively, in order to re-use the shuttering. There are two main beams (*Fig.* 17) with slightly tapered stems 16 in. thick in the spans and 22 in. thick above the piers. The 5-in. slab is carried by transverse beams prestressed with Leoba cables and cantilevering nearly 8 ft. 6 in. outside the main beams. The casings for the prestressing cables were supported by precast brackets, bolted to the shutters. The prestressing cable is spliced by overlapping loops to allow for the construction joint between the two parts of the bridge. The cable was arranged in four loops to reduce the local stresses at this anchorage. By this means forces of 3600 tons were carried over the construction joint, making the superstructure continuous for its full length.

"Concrete & Constructional Engineering."

Increase of Price.

The price of this journal has remained unchanged at 18*s*. a year for forty years, despite the fact that the cost of production has multiplied by four. It is now necessary to increase the price only because of the action of the Government in doubling the postage rates on periodicals in January last. At a time when the Government is urging the desirability of "price stabilisation," it is disturbing to find that its own refusal to do so forces others to increase prices. It is with much reluctance that it is necessary to pass this extra postage on to the readers of the journal by increasing the price per copy to 1*s*. 9*d*. and the annual subscription rate to 21*s*. starting with this number. Existing subscribers will not be asked to pay the increased rate until their subscriptions are due for renewal.

Book Reviews.

"Handbook of Engineering Materials." Edited by D. F. Miner and J. B. Seastone. (London: Chapman & Hall, Ltd. 1955. Price £7.)

THIS comprehensive handbook comprises a section on the selection of materials for all branches of engineering together with tables of mathematical and physical constants, and three sections dealing with the various materials of construction. The last three sections describe the physical, chemical, electrical, and thermal properties of many materials, with notes on their suitability for various manufacturing processes and their availability in the U.S.A. The intention is to present sufficient information to enable comparisons to be readily made between the materials that may be used for a particular purpose and to give sources of more detailed descriptions of each material. This appears to have been done in an admirable manner, and no substance of possible use in engineering seems to be omitted. The specifications quoted are generally those of the American Society for Testing Materials, but this does not detract materially from the value of the book to readers in other countries.

"Cálculo de Concreto Armado." Vol. 1. By T. van Langendonck. (Sao Paulo. 1954. The Brazilian Portland Cement Association. No price stated.)

THIS is the second edition of the first volume of a treatise, in the Portuguese language, on reinforced concrete. It is in three parts the first of which deals comprehensively with the strength of materials as applied to any elastic material; in the second part the properties of concrete and steel are considered together with a résumé of the requirements of the codes of practice of various countries and brief descriptions of some methods of prestressing; and the third part is entirely devoted to reinforced concrete members subjected to direct stresses. This last part is particularly interesting and includes methods for the calculation of stresses due to concentrations of load over part of a member, such as occur at the ends of prestressed members with post-tensioned cables. There are also notes on the design of some of the more well-known types of hinges in reinforced concrete, a subject which is frequently inadequately treated. Appendixes include tables of the geometrical properties and torsional resistances of various plane figures, bending moments and deflections of prismatic beams subjected to transverse loads and direct forces, coefficients of buckling for columns of various shapes, and tables for the design of columns. Those with a knowledge of the Portuguese language will find this book of value as it contains much information that is otherwise scattered in journals in many languages.

"Manuel du Béton Précontraint." By V. Weinberg. (Paris: Dunod. 1955. Price 2900 francs.)

BASED to a large extent on the requirements of the provisional regulations for prestressed concrete published in 1953 by the French Ministry of Public Works, this book outlines the theory of prestressed concrete with particular reference to systems using pre-tensioned wires. The methods are illustrated by several completely worked examples and by descriptions of the design and construction of a variety of structures. In addition the book includes methods for the rapid computation of bending moments on continuous beams, for the distribution of the effects on adjacent beams of a concentrated load applied to one or more beams in connected systems such as occur in the decks of bridges, for the calculation by M. Pigeaud's well-known method of bending moments due to concentrated loads on slabs, details of the French standard loads on bridges, and tables of the geometrical properties of box-shaped sections. The provisional French regulations of 1953 are given in full in an appendix. The book, in the French language, will be of interest to many because of its descriptions of the use of pre-tensioned wires in structures much larger than those to which the method is generally thought to be applicable.—J. E. G.

"Tragwerke aus Aluminium." By Fritz Stüssi. (Berlin: Springer-Verlag. Price 22.50 D.M.)

THIS book deals with the calculation, design, and construction of structures made of aluminium alloys. The tensile strengths of aluminium alloys are derived from data issued by the Northern Aluminium Company of Great Britain.

Analysis of a Parabolic Arch of Uniform Thickness.

By V. A. MORGAN, M.Eng., A.M.Inst.C.E.

It is often convenient to build arches with uniform depths as they are easier to construct than those of varying depth. The results of calculations for arches of uniform depth can be applied in the methods of design demonstrated in previous articles.

In this article general equations are derived which can be used in the design of arches of any rise-span ratio. The most usual case, where the rise is one-quarter of the span, is used here and the stiffness ratios, carry-over factors, arch thrust, etc., are derived for this ratio to simplify the calculations. The properties of arches with other rise-span ratios can be calculated from the equations derived.

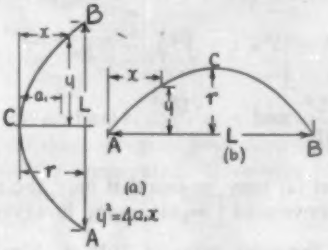


Fig. 1.

In some cases the properties are calculated to seven or more places of decimals; this is necessary for normal designs.

Two methods of integration can be used. For the parabolic arch shown in *Fig.* 1(*a*) the crown is assumed to be the origin and the rise of the arch is measured along the x axis. The equation of the parabola is

$$y^2 = 4a_1x \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

In *Fig.* 1(*b*) the left-hand springing is assumed to be the origin; the span L is measured along the x axis, and the equation of the parabola is

$$y = \frac{r}{L^2}(Lx - x^2) \quad . \quad . \quad . \quad . \quad . \quad (2)$$

In some cases integrations are derived from equation (1) and in others from equation (2); the result of one method can be found from the result of the other by changing the coefficients a and r. As an example, the length of the centre-line of a parabolic arch is derived from both equations and the results are compared.

The length of curve S measured along the centre-line calculated by the first method is

$$S = \int_{-S_1}^{S_1} dS = 2\int_0^{x_1} \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\, dx = 2\int_0^{x} \sqrt{1 + \frac{a_1}{x}}\, dx,$$

since $$\frac{dy}{dx} = \frac{2a_1}{y} = \frac{2a_1}{2\sqrt{a_1x}} = \sqrt{\frac{a_1}{x}}.$$

Substituting $x = a_1 \sinh^2\theta$, then $dx = 2a_1 \sinh\theta\cosh\theta$.

$$S = 4a_1\int_0^{\theta_1}\cosh^2\theta\, d\theta = 2a_1\int_0^{\theta_1}(1+\cosh 2\theta)\, d\theta = 2a_1\left(\theta + \frac{\sinh 2\theta}{2}\right)_0^{\theta_1}$$

$$= 2a_1\left(\sinh^{-1}\sqrt{\frac{x}{a_1}} + \sqrt{\frac{a_1x + x^2}{a_1^2}}\right)_0^{a_1r_1} = 2a_1(\sinh^{-1}\sqrt{r_1} + \sqrt{r_1 + r_1^2}) \quad . \quad (3)$$

By the second method the length of the curve is

$$\frac{dy}{dx} = \frac{4r}{L^2}(L - 2x) = Y. \quad dx = -\frac{L^2dY}{8r}.$$

$S = -\frac{L^2}{8r}\int_0^{Y_1}\sqrt{1+Y^2}\,dY.$ Substituting $Y = \sinh\theta$, then $dY = \cosh\theta$.

$$S = -\frac{L^2}{8r}\int_0^{\theta_1}\cosh^2\theta\, d\theta = -\frac{L^2}{16r}\int_0^{\theta_1}(1+\cosh 2\theta)\, d\theta = -\frac{L^2}{16r}\left(\theta + \frac{\sinh 2\theta}{2}\right)_0^{\theta_1}$$

$$= -\frac{L^2}{16r}\left(\sinh^{-1} Y + Y\sqrt{1+Y^2}\right)_{\frac{4r}{L}}^{-\frac{4r}{L}} = \frac{L^2}{8r}\sinh^{-1}\frac{4r}{L} + \frac{L}{2}\sqrt{1 + \left(\frac{4r}{L}\right)^2} . (4)$$

By substituting $a_1 = \frac{L^2}{16r}$ and $r_1 = \frac{16r^2}{L^2}$, equation (3) can be converted into equation (4).

As the answer from (4) may be found if only the rise and span are known it may often be more convenient : equation (3), however, is usually less cumbersome to use. For a rise-span ratio of 1 to 4, $S = \frac{L}{2}(\sinh^{-1} 1 + \sqrt{2})$; as $\sinh^{-1} 1 = 0{\cdot}8813735 87$, $S = 1{\cdot}147793575L$.

When tables of sinh x are not of sufficient accuracy, values may be calculated from $\sinh^{-1} x = \log_e(x + \sqrt{1+x^2})$.

By substitutions similar to those used to obtain equation (3) the following solutions are obtained :

$$2\int x\,ds = 2\int_0^{a_1r_1} x\sqrt{1+\frac{a}{x}}\,dx = \frac{a_1^2}{2}\{(1+2r_1)\sqrt{r_1^2+r_1} - \sinh^{-1}\sqrt{r_1}\} = 2\bar{x},$$

where $\bar{x}$ is a shortened notation for $\int x\,ds$. Let $\phi = \sinh^{-1}\sqrt{r_1}$, then

$$2\int x^2\,ds = \frac{a_1^3}{4}\{\phi + \tfrac{1}{3}\sqrt{r_1^2+r_1}(8r_1^2+2r_1-3)\} = 2\,.\,\bar{x}^2.$$

$$2\int a_1r_1 - x)\,ds = \frac{a_1^2}{2}\{(4r_1+1)\phi + (2r_1-1)\sqrt{r_1^2+r}\} = 2\,.\,\overline{a_1r_1 - x}.$$

$$2\int(a_1r_1 - x)^2\,ds = \frac{a_1^3}{4}\{(8r_1^2+4r_1+1)\phi + \tfrac{1}{3}(8r_1^2-10r_1-3)\sqrt{r_1^2+r}\} = 2\,.\,\overline{(a_1r_1-x)^2}.$$

$$\int x^2\sqrt{a_1x}\,ds = 2a_1^4\left\{\tfrac{1}{7}(1+r_1)^{\frac{7}{2}} - \tfrac{3}{5}(1+r_1)^{\frac{5}{2}} + \tfrac{1}{3}(1+r_1)^{\frac{3}{2}} - \frac{8}{105}\right\} = \overline{x^2\sqrt{a_1x}}.$$

$$\tfrac{1}{2}\int x^2(a_1r_1-x)\,ds = \frac{a_1}{128}\{(8r_1+5)\phi - \tfrac{1}{3}(15+14r_1-8r_1^2-16r_1^3)\sqrt{r_1^2+r}\} = \tfrac{1}{2}\overline{x^2(a_1r-x)}.$$

By similar substitutions to those used to obtain equation (4) the following solutions are obtained, within the limits of $Y_1 = \frac{4r}{L}$ and $Y_2 = \frac{4r}{L}(2n_1 - 1)$, where n_1L is the distance of vertical unit load from A.

$$\int_{n_1L}^{L} (x - n_1L)\, ds = \frac{L^3}{32r}\left\{(1-2n_1)(\phi_1 + Y\sqrt{1+Y^2}) - \frac{L}{6r}(1+Y)^{\frac{3}{2}}\right\}_{Y_2}^{Y_1} = \overline{x_A - n_1L}$$

where $\phi_1 = \sinh^{-1} Y$.

$$\int_{n_1L}^{L} x(x - n_1L)\, ds = \frac{L^4}{64r}\left\{(1 - 2n_1)\left[(\phi_1 + Y\sqrt{1 + Y^2}) + \frac{L}{3r}(1 + Y^2)^{\frac{3}{2}}\right]\right.$$
$$\left. + \frac{L^2}{64r^2}[Y(1 + 2Y^2)\sqrt{1 + Y^2} - \phi_1]\right\}_{Y_2}^{Y_1} = \overline{x_A(x_A - n_1L)}.$$

$$\int_{n_1L}^{L} y(x - n_1L)\, ds = \frac{L^3}{32}\left\{(1 - 2n_1)(\phi_1 + Y\sqrt{1 + Y^2}) - \frac{I}{6r}(1 + Y^2)^{\frac{3}{2}}\right.$$
$$\left. + \frac{L^2}{64r^2}(2n_1 - 1)[Y(1 + 2Y^2)\sqrt{1+Y^2} - \phi_1] + \frac{L^3}{32r^3}\left[\frac{(1+Y^2)^{\frac{5}{2}}}{5} - \frac{(1+Y^2)^{\frac{3}{2}}}{3}\right]\right\}_{Y_2}^{Y}$$
$$= \overline{y_A(x_A - n_1L)},$$

where x_A and y_A denote that these integrals have A as their origin. The former integrals are of use to obtain stiffness-ratios, carry-over factors, etc., whilst the latter are of use in constructing influence lines.

Horizontal Deflection at the Springings.

For convenience the axes x and y shown in *Fig.* 2 have been rotated through 90 deg. so that the span AB is horizontal. The origin is at C.

Consider the arch to be deflected horizontally by Δ at the springings. If the springings are considered to be hinged,

$$\int \frac{M(r_1a_1 - x)\, ds}{EI} = \Delta, \quad M = H_0(a_1r_1 - x), \text{ so that } 2H_0\int_0^{a_1r_1} (a_1r - x)^2\, ds = \Delta EI,$$

and

$$H_0 = \frac{\Delta EI}{2(a_1r_1 - x)^2} \qquad . \quad . \quad . \quad . \quad . \quad (5)$$

If the springings are constrained against rotation, $M = H(a_1r - x) - M_A$;

then $\int \frac{M\, ds}{EI} = 0$ and $\int \frac{M(a_1r_1 - x)\, ds}{EI} = \Delta$.

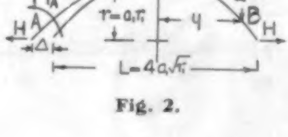


Fig. 2.

Therefore $$2M_A\int_0^{a_1r_1} ds = 2H\int_0^{a_1r_1}(a_1r_1 - x)\,ds$$

and $$2H\int_0^{a_1r_1}(a_1r_1 - x)^2\,ds - 2M_A\int_0^{a_1r_1}(a_1r_1 - x)\,ds = \Delta EI,$$

so that $$M_A = \frac{\overline{2a_1r_1 - x}EI\Delta}{4\,\overline{ds}\,\overline{(a_1r_1 - x)^2} - 4\overline{(a_1r_1 - x)}\,\overline{(a_1r_1 - x)}} \quad . \quad . \quad . \quad (6)$$

$$H = \frac{2\,\overline{ds}\,EI\Delta}{\text{denominator to } M_A} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

As $M_B = -M_A$ it can be shown, from equations (5), (6), and (7), that the general formula for the thrust, which applies to all arches of uniform thickness and for all conditions of fixity of the springings, is

$$H = H_o + \frac{\overline{2a_1r_1 - x}}{4\overline{(a_1r_1 - x)^2}}(M_A - M_B) \quad . \quad . \quad . \quad (8)$$

for a rise-span ratio of one to four. $a_1r_1 = r$, and $H_o r^3 = 0{\cdot}4395\ EI\Delta$,

$$M_A r^2 = -M_B r^2 = 1{\cdot}4739 EI\Delta, \text{ and } Hr^3 = 2{\cdot}325 EI\Delta.$$

Substituting in equation (8), $\dfrac{r^3H}{EI\Delta} = (0{\cdot}4395 + 1{\cdot}8855) = 2{\cdot}325$, since for this rise-span ratio $H = H_o + \dfrac{0{\cdot}6396}{r}(M_A - M_B)$.

Carry-over Factors and Stiffness Ratios.

SPRINGING B FIXED.—A bending moment M_A applied at springing A as in *Fig. 3a* produces a bending moment M_B at springing B; M_B divided by M_A is the carry-over factor. At any section D the bending moment is M, and

$$M = M_A - R_Ay_1 - H(a_1r_1 - x).$$

As C is the origin, x is positive and, from A to C, $y_1 = \dfrac{L}{2} - y$ and, from C to B $y_1 = \dfrac{L}{2} + y$. The fundamental equations to be solved are

$$\int_A^B My_1\,ds = 0 \qquad \text{and} \qquad \int_A^B M(a_1r_1 - x)ds = 0.$$

Substituting for M,

$$\int_0^{a_1r_1} M_A\left(\frac{L}{2} - y\right)ds + \int_0^{a_1r_1} M_A\left(\frac{L}{2} + y\right)ds - \int_0^{a_1r_1} R_A\left(\frac{L^2}{4} - Ly + y^2\right)ds$$
$$-\int_0^{a_1r_1} R_A\left(\frac{L^2}{4} + Ly + y^2\,ds\right) = \int_0^{a_1r_1} H\left[\left(\frac{L}{2} - y\right) + \left(\frac{L}{2} + y\right)\right](a_1r_1 - x)ds$$

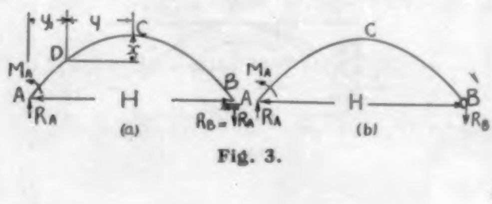


Fig. 3.

and, since $y^2 = 4a_1x$,

$$M_A - R_A\frac{L}{2} = \frac{2HL\overline{a_1r_1 - x} + 4R_A4\overline{a_1x}}{2L\,\overline{ds}}.$$

Also

$$2M_A\overline{a_1r_1 - x} - R_A\int_0^{a_1r_1}\left[\left(\frac{L}{2}-y\right)+\left(\frac{L}{2}+y\right)\right](a_1r_1 - x)ds = 2H\overline{(a_1r_1 - x)^2}.$$

Therefore
$$M_A - R_A\frac{L}{2} = \frac{2H\overline{(a_1r_1 - x)^2}}{2\overline{a_1r_1 - x}},$$

so that
$$R_A = \frac{HL.2\overline{ds}.2\overline{(a_1r_1 - x)^2} - 4HL.\overline{a_1r_1 - x}.\overline{a_1r_1 - x}}{16.\overline{a_1x}.2\overline{a_1r_1 - x}}.$$

$$H = \frac{16M_A\overline{a_1x}.2\overline{a_1r_1 - x}}{2\overline{(a_1r_1 - x)^2}[L^2\overline{ds} + 16\overline{a_1x}] - 2L^2\overline{a_1r_1 - x}.\overline{a_1r_1 - x}}.$$

Therefore

$$R_A = \frac{M_AL[2\overline{ds}\;2\overline{(a_1r_1 - x)^2} - 4\overline{a_1r_1 - x}.\overline{a_1r_1 - x}]}{\text{denominator of } H} \qquad . \quad . \quad (9)$$

M_B is obtained by calculating the moments of all external forces about B, and $M_A + M_B - R_AL = 0$. As springing B is fixed, the rotation θ_A at springing A due to M_A is

$$EI\theta_A = \int_A^B M\,ds = 2M_A\overline{ds} - R_AL\,\overline{ds} - 2H\overline{a_1r_1 - x}$$

$$= \frac{M_A16\overline{a_1x}[2\overline{ds}\;2\overline{(a_1r_1 - x)^2} - 4\overline{a_1r_1 - x}\;\overline{a_1r_1 - x}]}{\text{denominator of } H} \qquad . \quad (10)$$

Therefore $EI\theta_A = 16\overline{a_1x}.\dfrac{R_A}{L} = \dfrac{R_A}{L}.4a_1^3[(1 + 2r_1)\sqrt{r_1^2 + r_1} - \phi]$ . . (11)

For a rise-span ratio of 1 to 4, $a_1 = \dfrac{L^2}{16r} = \dfrac{L}{4}$ and $r_1 = \dfrac{16r^2}{L^2} = 1$.

Substituting in equation (9),

$$R_A = 0{\cdot}6814\frac{M_A}{L} \quad \text{and} \quad M_B = -\,0{\cdot}3186M_A,$$

so that the carry-over factor is $-\dfrac{1}{3{\cdot}14} = \dfrac{M_B}{M_A}$.

$$EI\theta_A = 0{\cdot}6814\frac{M_A}{L^2}.\frac{L^3}{16}.3{\cdot}3592 = 0{\cdot}14305M_AL.$$

The stiffness of the arch at springing A is equal to the bending moment at A producing unit rotation at A. Therefore $\theta_A = 1 = 0{\cdot}14305M_A\dfrac{L}{EI}$ and $S = M_A = 6{\cdot}99E\dfrac{I}{L} = 7EK$ approximately, where S is the stiffness ratio, and $K = \dfrac{I}{L}$.

SPRINGING B HINGED.—From equation (8), and as H_o and M_B are zero (*Fig. 3b*),

$$H = \frac{\overline{2a_1r_1 - x}}{4(a_1r_1 - x^2)} M_A \text{ and for } \frac{r}{L} = \frac{1}{4}, H = \frac{0{\cdot}6396}{r} M_A.$$

In this case $\theta_A L$ is equal to the deflection at springing B relative to the tangent of springing A, and θ_A is the angle between the tangents of the arch at springing A before and after M_A is applied. $\theta_A L$ is equal to the moment of the area of the bending moment, the distance at which this acts being measured along the centre-line of the arch from B.

$$EIL.\theta_A = \int_A^B M_A(L - y_1)ds - \int_A^B R_A(L - y_1)y_1\,ds - \int_A^B H(L - y_1)(a_1r_1 - x)ds.$$

Substituting $y_1 = \frac{L}{2} - y$ and $\frac{L}{2} + y$ from A to C and C to B respectively,

$$EIL.\theta_A = \frac{L}{2} M_A.2\overline{ds} - R_A \frac{L^2}{4}.2\overline{ds} + R_A 8\overline{a_1x} - H\frac{L}{2}.2\overline{a_1r_1 - x} \text{ and } R_A = \frac{M_A}{L}.$$

$$EI\theta_A = \frac{M_A}{4}\left[2\overline{ds} + \frac{32\overline{a_1x}}{L^2} - \frac{\overline{2a_1r_1 - x}.\overline{a_1r_1 - x}}{(a_1r_1 - x)^2}\right], \text{ and if } \frac{r}{L} = \frac{1}{4},$$

$$EI\theta_A = M_A\frac{L}{16}[4{\cdot}5912 + \tfrac{1}{2} \times 3{\cdot}359 - 0{\cdot}6396 \times 5{\cdot}8211] = 0{\cdot}1592 M_A L,$$

so that, for $\theta_A = 1$, the stiffness $S = M_A = 6{\cdot}28EK$.

Uniform Arches with Side Loads.

BOTH SPRINGINGS FIXED.—For a side load acting uniformly on the vertical projection of the arch (*Fig. 4a*) the bending moment between A and C is

$$M = M_A + R_A y_1 - H_1(a_1r_1 - x) = M_A + R_A\left(\frac{L}{2} - y\right) - H_1(a_1r_1 - x)$$

and for a uniformly-distributed load acting on the vertical projection of the arch between C and B is

$$M = M_A + R_A\left(\frac{L}{2} + y\right) - H_1(a_1r_1 - x) - \frac{wx^2}{2}.$$

Substituting in the fundamental equations

$$\int_A^B M\,ds = 0, \quad \int_A^B M(a_1r_1 - x)ds = 0, \quad \text{and} \quad \int_A^B M(L - y_1)ds = 0,$$

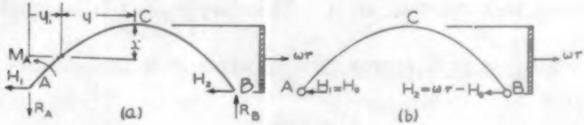


Fig. 4.

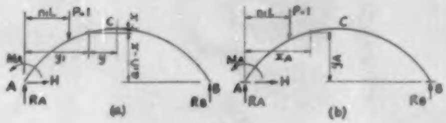


Fig. 5.

the values for M are

$$(2M_A + R_AL)\overline{ds} = 2H_1\overline{a_1r_1 - x} + \frac{w}{2}\overline{x^2} \quad . \quad . \quad . \quad (12)$$

$$(2M_A + R_AL)\overline{a_1r_1 - x} = 2H_1\overline{(a_1r_1 - x)^2} + \frac{w}{2}\overline{(a_1r_1 - x)x^2}. \quad . \quad (13)$$

$$(2M_A + R_AL).\frac{L}{2}.\overline{ds} = H_1L\overline{a_1r_1 - x} + R_A.8\overline{a_1x} + \frac{wL}{4}\overline{x^2} - w\overline{x^2\sqrt{a_1x}}\ . \quad (14)$$

Multiplying equation (12) by $\frac{L}{2}$ and subtracting it from equation (14),

$$R_A8\overline{a_1x} = w\overline{x^2\sqrt{a_1x}} \quad \text{so that} \quad R_A = \frac{w\overline{x^2\sqrt{a_1x}}}{8\overline{a_1x}} \quad . \quad . \quad (15)$$

Also

$$H_1 = \frac{w}{4}\cdot\frac{\overline{(a_1r_1 - x)}\,\overline{x^2} - \overline{ds}\,\overline{(a_1r_1 - x)x^2}}{\overline{(a_1r_1 - x)^2}\,\overline{ds} - \overline{a_1r_1 - x}\,\overline{a_1r_1 - x}} \quad . \quad . \quad (16)$$

and

$$M_A = H_1\frac{\overline{a_1r_1 - x}}{\overline{ds}} + \frac{w}{4}\frac{\overline{x^2}}{\overline{ds}} - R_A\frac{L}{2} \quad . \quad . \quad . \quad (17)$$

For a rise-span ratio of 1 to 4,

$R_A = 0{\cdot}06552wr,\quad H_1 = 0{\cdot}2198wr,\quad H_2 = 0{\cdot}7802wr,$

$M_A = 0{\cdot}06524wr^2,$ and $M_B = 0{\cdot}17268wr^2.$

Both Springings Hinged.—For this case (*Fig.* 4*b*), as the horizontal deflection of the hinges is zero, by substituting $M_A = 0$ in (13),

$$H_1 = H_o = \frac{2R_AL\overline{a_1r_1 - x} - w\overline{x^2(a_1r - x)}}{4\overline{(a_1r - x)^2}}.$$

If $L = 4r,\ H_o = 0{\cdot}28855wr$ and $H_2 = wr - H_o.$

By substituting these values in equation (8),

$$\frac{H_1}{wr} = 0{\cdot}28855 + 0{\cdot}6396(0{\cdot}06524 - 0{\cdot}17268).$$

This checks the previous value $H_{1\,(\text{fixed})} = 0{\cdot}21983wr.$

Influence Line for Load Moving across the Arch.

The integrals already determined with C as the origin may be used in calculating the integral coefficients of M_A, R_A, and H ; however, it is more convenient to use A as the origin for integrals involving P as in *Fig.* 5(*b*). Then for springings A and B considered as fixed,

$$M = M_A - R_Ay_1 + H(a_1r_1 - x) + P(x_A - n_1L).$$

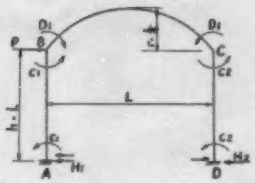


Fig. 6.

$$\int_A^B M\,ds = 0 = 2M_A\overline{ds} - R_A L\,\overline{ds} + 2H\,\overline{a_1r_1 - x} + P\,\overline{x_A - n_1L} \quad . \quad (18)$$

$$\int_A^B M(a_1r_1 - x)ds = 0$$

$$= 2M_A\overline{(a_1r_1 - x)} - R_A L\,\overline{a_1r_1 - x} + 2H\overline{(a_1r_1 - x)^2} + Py_A\overline{(x_A - n_1L)} \quad . \quad (19)$$

$$\int_A^B M\,(L - y_1)ds = 0$$

$$= (2M_A - R_A)\frac{L}{2}\,\overline{ds} + R_A 8\overline{a_1x} + HL\overline{a_1r_1 - x} + Px_A\overline{(x_A - n_1L)} \quad . \quad (20)$$

By substituting fractional or decimal span-ratios for n_1, such as 0·1 to 0·9, equations (18) to (20) may be solved for any rise-span ratio and the influence lines drawn. If the springings are hinged, R_A is obtained by calculating moments of external forces about B and substituting $M_A = 0$ in equation (19). For this condition of the springings

$$H = \frac{P[(1 - n_1)L\,.\,\overline{a_1r - x)} - y_A\overline{(x_A - n_1L)}]}{\overline{(a_1r_1 - x)^2}}.$$

Example.

The various factors and ratios for a parabolic arch of uniform thickness can be applied in the manner used for arches, the moments of inertia of which vary as the secant of the angle of the arch. The procedure in this example may be used for other conditions of loading.

Consider an arch with a force P applied at the springings (*Fig.* 6). The properties assumed are $r = \frac{L}{4}$; height of columns $h = L$. The moments of inertia of the arch and column I_a and I_c are both equal to I. Δ_1 and Δ_2 are deflections at springings B and C, and C_1, D_1 and C_2 are fixing moments produced at B and C by P.

$$C_1 = \frac{6I_C\Delta_1E}{h^2} = \frac{6IE\Delta_1}{L^2}. \qquad C_2 = \frac{6IE\Delta_2}{L^2}. \qquad \text{Then for } L = 4r,$$

S_a = arch stiffness ratio $= \frac{7EI}{L}$, the arch carry-over factor is $-\frac{1}{3 \cdot 14}$.

S_c = column stiffness ratio $= \frac{4EI}{L}$ and the column carry-over factor is $\frac{1}{2}$.

TABLE I.

A c.o. ½		B c.o. $-1/3{\cdot}14 = -0{\cdot}3186$		C c.o. ½	D
	4	7	7	4	
C_1	C_1	$-D_1$	D_1	C_2	C_2
	$\frac{4}{11}(D_1-C_1)$	$\frac{7}{11}(D_1-C_1)$	$-\frac{7}{11}(D_1+C_2)$	$-\frac{4}{11}(D_1+C_2)$	
$\frac{2}{11}(D_1-C_1)$		$+\frac{2{\cdot}23}{11}(D_1+C_2)$	$-\frac{2{\cdot}23}{11}(D_1-C_1)$		$-\frac{2}{11}(D_1+C_2)$
	$-\frac{4\times 2{\cdot}23}{121}(D_1+C_2)$			$+\frac{4\times 2{\cdot}23}{121}(D_1-C_1)$	
$-\frac{2\times 2{\cdot}23}{121}(D_1+C_2)$ etc.				etc.	$+\frac{2\times 2{\cdot}23}{121}(D_1-C_1)$

$$D_1 = \frac{1{\cdot}4739}{r^2}EI(\Delta_1 - \Delta_2) = 3{\cdot}9304(C_1 - C_2).$$

$$H_o = \frac{0{\cdot}4395}{r^3}EI(\Delta_1 - \Delta_2) = \frac{1{\cdot}172}{r}(C_1 - C_2).$$

The coefficients of the terms in *Table* 1 decrease in geometrical form, their ratio being $\left(\frac{2{\cdot}23}{11}\right)^2$. The sum of the terms distributed to infinity is the first term of $(D_1 - C_1)$ and $(D_1 + C_2)$ in column A multiplied by $\dfrac{1}{1-\left(\frac{2{\cdot}23}{11}\right)^2} = 1{\cdot}045$.

By substituting $3{\cdot}9304(C_1 - C_2)$ for D_1 and multiplying by $1{\cdot}045$:

$$M_{AB} = C_1 + \left[\frac{2}{11}(D_1 - C_1) - \frac{2}{121}\times 2{\cdot}23(D_1 + C_2)\right]1{\cdot}045 = 1{\cdot}408C_1 - 0{\cdot}637C_2.$$

Similarly

$M_{BA} = 1{\cdot}816C_1 - 1{\cdot}274C_2 = -M_{BC}$, $M_{CB} = 1{\cdot}274C_1 - 1{\cdot}816C_2 = -M_{CD}$, $M_{DC} = -0{\cdot}637C_1 + 1{\cdot}408C_2$, $LH_1 = M_{AB} + M_{BA} = 3{\cdot}224C_1 - 1{\cdot}911C_2$,

and $LH_2 = M_{CD} + M_{DC} = -1{\cdot}911C_1 + 3{\cdot}224C_2$.

$$-H_2 = -H_o + \frac{0{\cdot}6396}{r}(3{\cdot}090C_2 - 3{\cdot}09C_1)$$

$$= \frac{4{\cdot}688}{L}(C_2 - C_1) + \frac{7{\cdot}9}{L}(C_2 - C_1),$$

so that $LH_2 = 12{\cdot}588(C_1 - C_2) = 3{\cdot}224C_2 - 1{\cdot}911C_1$.

Therefore $C_1 = 1{\cdot}093C_2$ and, as $P = H_1 + H_2$, $C_1 = 0{\cdot}398PL$, and $C_2 = 0{\cdot}364PL$.

By substituting these values of C_1 and C_2,

$M_{AB} = 0{\cdot}328PL$, $M_{BA} = 0{\cdot}259PL$, $H_1 = 0{\cdot}587P$,

$M_{CD} = 0{\cdot}155PL$, $M_{DC} = 0{\cdot}258PL$ and $H_2 = 0{\cdot}413P$.

By calculating moments about A, $R_B = -R_A = 0{\cdot}414P$.

As shown in previous articles these moments and thrusts are of value in calculating side thrusts and in designing frames subjected to any other loading conditions.

Failure of a Warehouse.

THE requirements of codes of practice in relation to the actual strength of structures were discussed by Mr. Boyd G. Anderson (an associate partner of Messrs. Ammann & Whitney, consulting engineers) in a paper presented to the annual convention of the American Concrete Institute in February last. The following is an abstract of the paper.

The design of concrete members to resist shearing forces and diagonal tension is not fully understood, and in some cases designing according to a code of practice has not prevented failures. The major differences between American codes and those of other countries are that the United States and Canadian codes assume that the concrete will always participate in resisting shearing forces while most European codes require that the whole of the shearing force be resisted by reinforcement if the permissible shearing stress of the concrete is exceeded. The German code further requires that reinforcement to resist shearing forces must be provided throughout the span. Most of the European codes relating to plain bars recommend the use of nominal stirrups throughout a member, and recommend the bending up of as much of the longitudinal reinforcement as is practicable.

The codes do not recognise possible principal stresses caused by combinations of shearing and high bending stresses, or the effect of volumetric changes, or any other cause of axial tension or compression in the member. Nor do they recognise the possibility that bending stresses may be needed in combination with shearing stresses to produce the simultaneous participation of concrete and reinforcement that is assumed by present design methods.

Strict dependence on codes has resulted in an overwhelming number of properly-functioning structures, but experience has indicated that the designer should not rely completely on the minimum requirements of codes.

For example, three bays recently collapsed in a warehouse constructed under the direction of the Office of Chief of Engineers for use by the U.S. Air Force. The warehouse was 400 ft. wide and 2000 ft. long with beams continuous over six spans. The building was separated by an expansion joint near the centre of the width and by double frames of short-span at 200-ft. intervals along the length. The six spans were about 67 ft. long and were formed of haunched rigid frames. The collapse occurred in three bays of a penultimate span. Severe cracking had been observed in one of the frames prior to failure, and this bay was supported by temporary shoring but it later failed when the adjacent bays collapsed. The failure occurred about 1 ft. 6 in. beyond the position at which the reinforcement to resist the negative bending moment was curtailed and the calculations indicated a small positive bending moment.

Inspection indicated that the failure was the result of a general weakness resulting in cracking in the same span throughout most of the building. The cracks generally developed about a year after the application of the full dead load, thus indicating that shrinkage and creep may not have been fully taken into account. It appeared that the quality of the concrete and reinforcement was in accordance with the specification and, according to tests, had greater strength than that assumed in the design.

A study of the building and of test-beams made to one-third scale suggested that the provision of reinforcement to resist shearing throughout the length of the member, as required by some codes, would have resisted the shearing forces, but cracking might still have occurred unless a sufficient portion of the reinforcement resisting the negative bending moment was extended farther than is required by the usual elastic analysis. No codes, either American or European, contain provisions that would have prevented the failure.

Standard for Sands for Plastering.

BRITISH Standard No. 1198, just issued, relates to sands for internal plastering with gypsum plasters (with or without the addition of lime).

Joints in Precast Frames.

By C. H. DOBBIE, B.Sc., M.I.C.E., and R. L. WAJDA, Dipl.Ing., A.M.I.C.E.

JOINTS are necessary in precast arch frames in order that the parts may be of convenient size and weight. In the design of such joints in a comparatively new material it is natural to seek guidance from methods used for older materials and which have stood the test of time. Timber scarf joints, as in walings (*Fig.* 1) of groynes, have been adapted to reinforced concrete in this manner. Scarf joints for timber have been used since 2000 B.C. and are mentioned in the Naval Accounts of Henry VII in 1497. The ends were cut to interlocking shapes and held together by central hardwood wedges and hardwood cramps in tapering holes, and more recently by steel bolts, sometimes with fishplates. Steel bolts are used for concrete units, recesses being provided so that bolt-heads and nuts may be grouted to prevent corrosion.

The following notes describe research on this type of joint undertaken by R. L. Wajda at Battersea Polytechnic by permission of Mr. W. J. Peck, M.Sc., M.I.Mech.E., Head of the Department of Civil and Mechanical Engineering, and assistance was given by Mr. V. C. Davies, B.Sc., M.I.Mech.E., who has an overall responsibility for research work at the College.

Common forms of joint used in precast concrete are shown in *Fig.* 2 (a) and (b). The scarf joint tested (*Fig.* 3) differs in that it is to some extent self-locking, due to the notched projections on the sloping faces. This joint was designed to connect the parts of a reinforced concrete precast gable frame (*Fig.* 4) of 41 ft. 4 in. span. The joints were provided at the estimated points of contraflexure of the sloping part when carrying dead load only, and were 6 ft. 3 in. from the centres of the vertical members (measured horizontally). The bending moment at the joint due to dead load and the wind was estimated to be about $1\frac{1}{2}$ ft.-tons.

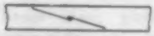


Fig. 1.

Fig. 2.

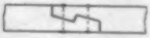


Fig. 3.

The concrete used was a $1 : 1\frac{1}{2} : 3\frac{1}{2}$ nominal mixture with an aggregate of $\frac{3}{4}$ in. maximum size and a water-cement ratio of 0·42, and was compacted by an electric hammer weighing $17\frac{1}{2}$ lb. and imparting 1600 blows a minute. The compressive strength of test cubes made at the factory was from 5000 to 5500 lb. per square inch at seven days and 7000 to 7500 lb. per square inch at 28 days, but cubes made from a small mixture made for the specimen joints had a crushing strength of only 3600 lb. per square inch

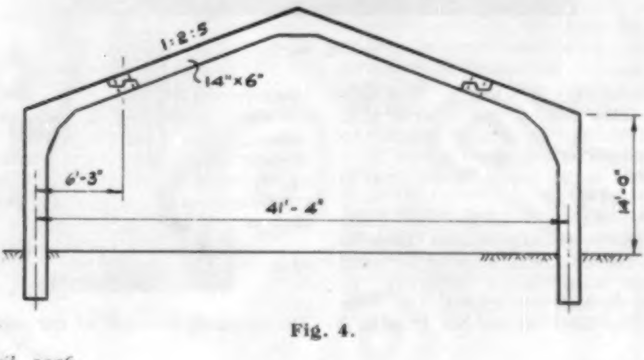


Fig. 4.

at 28 days. As the tests made with this concrete were satisfactory the results are published, but it is intended to make further tests with concrete as used in the work.

The following tests were made, (1) bending only, (2) shearing, (3) direct thrust and bending. The tests were made on a compression machine of 50 tons capacity; the arrangement of the tests are shown in *Figs.* 5, 6 and 7. The supports in tests Nos. 1 and 2 were 5 ft. apart and the load was applied symmetrically at two points 3 ft. apart in test No. 1 (*Fig.* 8) and at the centre of the joint in test No. 2 (*Fig.* 9).

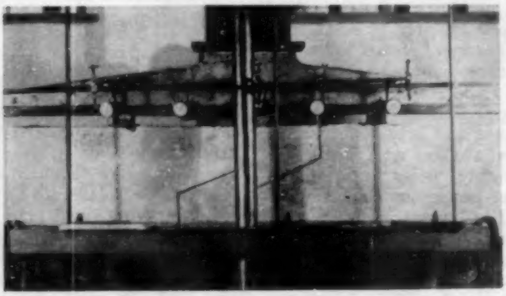


Fig. 5.

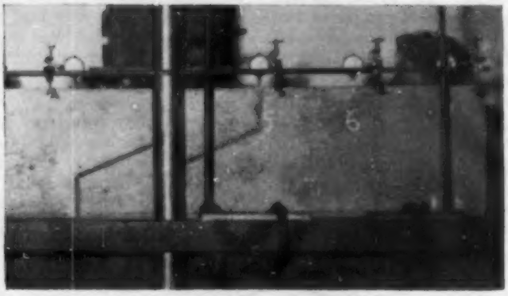


Fig. 6.

Measurements of deflection were made at five points equally spaced between supports, the load being applied by small increments until failure occurred. In test No. 3 the load was applied 3 in. from the axis (*Fig.* 10). In test No. 1 the load causing failure of the joint by crushing of the concrete in the compressive zone was 22 tons, which produced a uniform bending moment of 11 ft.-tons over the central 3 ft. of the specimen.

An estimate of the bending moment causing failure of the corresponding full section of a reinforced concrete beam can be obtained from $M_{ult.} = 0{\cdot}33\ bd^2c$, in which b is the breadth of the section, d the effective depth, and c the strength of cylinders (about 0·8 of the strength of cubes). Thus the ultimate bending moment of a reinforced concrete section 14 in. by 6 in. with concrete having a crushing strength of 3600 lb. per square inch is

$$M_{ult.} = \frac{0{\cdot}33 \times 6 \times 13^2 \times 3600 \times 0{\cdot}8}{2240 \times 12} = 35{\cdot}8 \text{ ft.-tons.}$$

The breaking strength of the joint tested

was thus 30·8 per cent. of the breaking strength of the beam.

The maximum deflection at the centre of the joint at the moment of failure was ½ in., and the relation between the load and this deflection is shown in *Graph* No. 1. The almost perfect linear relationship in the first part of the curve indicates the elastic behaviour of the joint up to about 6½ tons load and is attributed to the mortar in the joint. The change of slope corresponds to the load at which opening of the joint on the tensile side was first noted, and may be attributed to the failure of the mortar at the bottom of the joint. The opening of the joint reduced its stiffness, and, on increasing the load beyond 6½ tons, yielding was observed up to a load of about 12 tons. With greater loads the relation became linear again up to failure. Yielding of the joint at loads between 6½ tons and 12 tons may be attributable to the opening of the joint at the bottom when the load was 6½ tons, and the gradual transfer of the moment of resistance to the compressive zone and to the bolts; this stage of loading corresponds to the curved part of the load-deflection diagram. At the end of this stage a hair crack had been formed at the point of the notched projection in the joint, which shows that the self-locking effect was considerable. The linear relationship between load and deflection

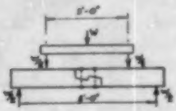


Fig. 8.

Fig. 9.

Fig. 10.

beyond 12 tons and to failure probably indicates that the combined strength of the concrete in compression and the bolts in tension was effective until crushing of the concrete occurred.

The shearing force at the joint was estimated to be about 3 tons. In test No. 2 the shearing force at the joint at the moment of failure was 5 tons, produced by a load of 10 tons at the centre, but the failure at that load was caused by crushing of the concrete in the compressive zone and not by shearing. The load factor for shearing was then $\frac{5}{3} = 1{\cdot}67$. The bending moment that caused failure of the joint was $5 \times 2{\cdot}5 = 12{\cdot}5$ ft.-tons, which is 10 per cent. greater than the bending moment causing failure in test No. 1. In test No. 2 the strength of the joint in bending was about 35 per cent. of that of a monolithic reinforced concrete beam of the same dimensions. The load-deflection curves for this method of loading are shown in *Graph* No. 2. There was a deviation from the linear relationship in the curves at all sections measured, and the form of the curve for point 4 (the centre of the joint) in which, after considerable yielding, the joint appeared to stiffen (between loads of 4½ and 7½ tons) and then recommenced normal yielding, indicated that in this case the bolts had not been tightened effectively.

In test No. 3 the load was applied at an eccentricity of 3 in., which corresponds well to the internal forces in the girder of a gable frame. (The maximum direct force in the girder had been estimated to be about one ton.) The direct load causing failure was 52 tons, which produced a bending moment of $52 \times 0{\cdot}25 = 13$ ft.-tons. In this test again the failure of the joint was due to crushing of the concrete

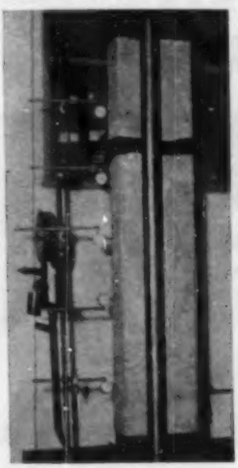
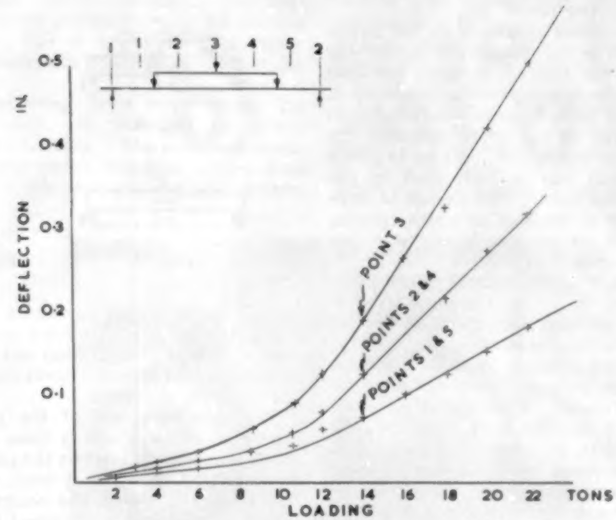
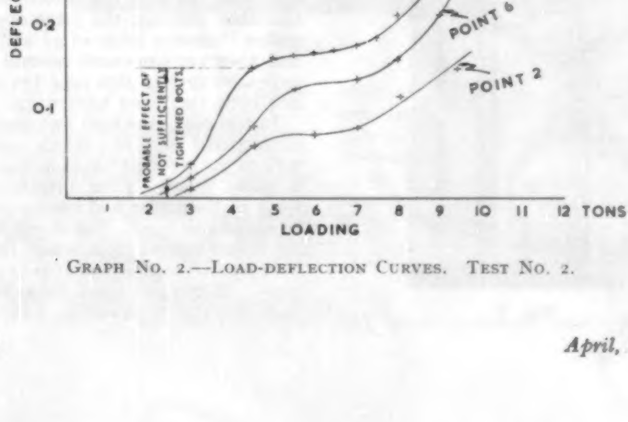


Fig. 7.



GRAPH NO. 1.—LOAD-DEFLECTION CURVES. TEST NO. 1.



GRAPH NO. 2.—LOAD-DEFLECTION CURVES. TEST NO. 2.

Fig. 11.

at both ends of the joint, which is probably the result of a small eccentricity that created compressive stresses over the whole section of concrete. The effect of bending was comparatively small in this test, as is proved by the small deflection of 0·2 in. at failure.

Comparing the bending moment caused by the load at failure and the ultimate bending moment of a reinforced concrete section of the same dimensions, the strength of the joint may be assumed to be 36 per cent. of the strength of the monolithic section.

The conclusions derived from the tests are : (a) The joint can be considered to be entirely satisfactory for connecting precast members at their points of minimum bending moment. (b) The load causing failure of the joint was different for different methods of loading.

In test No. 1 an ultimate load of 22 tons caused a bending moment of 11 ft.-tons. In test No. 2 an ultimate load of 10 tons caused a bending moment of $12\frac{1}{2}$ ft.-tons. In test No. 3 an ultimate load of 52 tons caused a bending moment of 13 ft.-tons. The estimated value of the ultimate bending moment of a monolithic reinforced concrete section of the same dimensions (14 in. × 6 in.) was 35·8 ft.-tons, so that the strength of the joint may be assumed to be 30 per cent. to 35 per cent. of the strength of a monolithic section of the same dimensions.

(c) The joint was able to resist a bending moment equal to about 30 per cent. of that resisted by a monolithic section of the same dimensions. The load factors of the joint and a monolithic section were the same.

(d) The failure of the joint was due to crushing of the concrete ; the strength of the joint can, however, be increased by using stronger concrete.

(e) The measured deflections may be considered consistent and satisfactory.

(f) The scarf joint, by providing a measure of interlocking, considerably increases the overal strength of the joint.

(g) It is very necessary to tighten the bolts effectively.

Generally the tests showed that the joint was rigid and that the projection increased its strength and rigidity. The results indicate that large bending

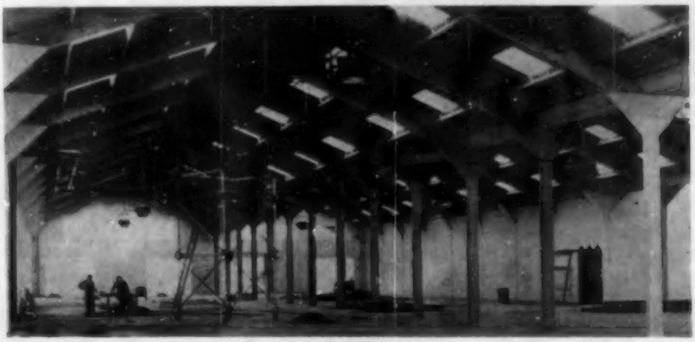


Fig. 12.

moments can safely be resisted by the joint, so that there is some small freedom of choice of the position of the joint. Further tests are proposed.

Fig. 12 shows frames with this type of joint in a structure built at Coleshill, near Birmingham, for Imperial Chemical Industries, Ltd.

[Mr. C. H. Dobbie was the consulting engineer for the structure at Coleshill (*Fig.* 12), and the frames were made by Precast Utilities (London), Ltd.]

The British Standard for Aggregates.

WE have received the following from Mr. Dudley P. Harris, of Ruislip, Middlesex.

I was very surprised to read in the penultimate paragraph of the Editorial Note in your February number disparaging references to the recent revision of the British Standard for Concrete Aggregates, which you say " permits the use of sands which were not previously permitted, and which would have been rejected by many engineers ". This is a surprising statement to be made by what is regarded as our foremost and enlightened journal on concrete in this country.

The facts about sand are too well known and make it unnecessary for me to repeat that equally satisfactory concrete can be produced with all the sand gradings referred to in the revised B.S. If some of the sands to which you refer were rejected by engineers in the past, one can only attribute this to the fundamental lack of understanding which has existed so long on the various inter-related factors which go to make concrete of different qualities and properties, and it is in keeping with the times that this revision has become essential, regardless of the fact that original gradings are slowly becoming short in supply.

Gap-graded aggregates are being used increasingly and with success in normal structural concrete, and are regarded as an essential feature in major hydraulic undertakings where large-size aggregates are used. The implications of this are clearly related to the inter-relationship of particle sizes and must be considered on their own merits, but this surely cannot be regarded as the lowering of a standard, but rather indicates a better understanding.

[In the revised Standard sands in four zones of varying degrees of fineness replace sands in classes A and B of the previous edition, and sand in zone 4 can contain 10 per cent. more passing a No. 50 sieve than a sand in the former class B. As we stated, many engineers would not have used such very fine gradings if better were available. This was apparently realised by the compilers of the Standard, for it is stated in the foreword that sand in zone 4 should not be used in reinforced concrete until tests have been made to ascertain its suitability. This warning is given because it is not so easy to make good concrete with some of the sands in zones 1 and 4 as it is with more uniformly-graded sands of medium fineness. We suggested that this was a standard of expediency, and that this was realised by the compilers of the Standard is shown in an explanatory statement issued with the Standard in which it was stated that where only very fine or very coarse sands are available at a reasonable cost they must be used. If engineers were satisfied that such sands were suitable they would have used them without exhortations to do so from the British Standards Institution. There is no doubt that by careful proportioning such sands will make good concrete but, as the Standard suggests by its recommendation that tests be made before sand in zone 4 be used in reinforced concrete, it is fairly obvious that engineers who prefer sands of medium fineness moduli rather than sands with a high proportion of one size will have less confidence in the four new zones than in the previous two classes. It may also be mentioned that the general specification of the Ministry of Works prohibits the use of sand in zone 4 without the permission of the Ministry. —ED.]

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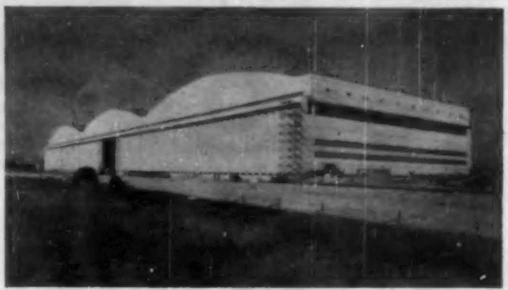

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The Durability of Concrete.

THE following notes are abstracted from a paper read by Mr. T. Whitley Moran recently at the Institution of Structural Engineers. The paper gives much detailed information on the use of gunite in the repair of deteriorated concrete structures of many types. The paper is given in full, with illustrations, in "The Structural Engineer" for February, 1956.

The earliest reinforced concrete structures were built in this country about 1897 and the total number of structures built by, say, 1910 was not very high. By about 1920 reinforced concrete was fully established, and many designers and contractors undertook such work. Contracts began to be placed on the basis of competitive design and on the lowest tender irrespective of experience. Margins of cover on the steel were sometimes reduced, and as a result many structures built in the past thirty-five years have proved less durable than their predecessors.

By 1925–30 a few of the earliest British structures had begun to show serious deterioration and spalling of the concrete. The normal type of deterioration is essentially due to rusting of the reinforcement as a result of inadequate cover or porous concrete, or both. The lack of cover may be due to careless placing of the reinforcement or to bad design. Damage due to frost and the chemical attack of sea-water are seldom critical unless the quality of concrete is very bad.

Initially repairs took the form of patching with the trowel, a useful but short-term expedient. Later, and where the damage was more severe, a covering of fresh concrete was cast on behind shutters. This method proved to have a limited life, as little bond was obtained, atmospheric moisture was not kept from the reinforcement, the deterioration continued, and patches of concrete usually fell off after some years. It was in these circumstances that American practice was copied and gunite began to be used.

Unfortunately there seems to be no indication that recent structures are any more durable than those built in the first decade of this century. In a paper read in 1937 the author said, "It will be appreciated that defects of the type described above are rarely met with in modern reinforced concrete structures. Present-day designers allow more cover over the reinforcement, and contractors are more experienced in the placing of reinforcement and in the precautions to be observed in forming the concrete." These statements were premature and have since proved to be only partly true. Far too little attention is paid to durability. Many structures now have to be repaired after a life of less than twenty years. There is a vast literature on refinements in design and in construction, but little on durability, and very little information has been recorded on the practical aspects of the deterioration and repair of concrete structures.

In 1925 interest was mainly concentrated on the cement gun itself, and it was recognised as a useful tool for refacing concrete. Later, attention shifted from the apparatus to the method of using the process. By 1935 investigations had moved on to the physical properties of gunite, and the various means of utilising them. The repair of reinforced concrete structures was still largely regarded as a maintenance operation to be applied locally to defective areas, but attention is now being gradually transferred from the gunite to the structure on which it is proposed to use it.

Consider, for example, a large warehouse which has much external deterioration but is in very good order internally and which would cost £250,000 to replace. The outside might be scaffolded and patched at a cost of £5000, but it would later have to be patched again. On the other hand it would be possible completely to recondition the building with gunite for a sum of £15,000 or £20,000 so that it would be sound for forty or fifty years. Similarly a wharf or bridge might be renovated for about one-fifth of the cost of reconstruction.

There seems to be no reason why the average reinforced concrete structure should not have a life of a century if it were completely re-surfaced, say twice, in that period. This conception of the complete renewal and refitting of old concrete structures is far removed from the ideas of twenty-five years ago, and is by

far the most important structural use of gunite to-day.

By 1937 several clear principles had been discovered when assessing repair problems in which there were genuine structural defects. For example, dealing with water is an engineering problem and the remedy must be applied on the water face if it is to have any permanent value. Sometimes this can be done by pressure grouting, but often it is better to devise means of underdrainage or of leading the ground-water away downhill or to a sump. It is absolutely essential to diagnose the cause of the trouble accurately, otherwise the remedy may be completely ineffective. If a structure is leaking through cracks, the cause of the cracks must be ascertained and if possible steps taken to prevent the cracks recurring. Until this is done it is futile to try to stop the leak by plugging the cracks; that would be merely treating the symptoms and not the disease.

After the diagnosis has been made and confirmed by borings or tests, the problem presents two separate aspects, namely the repair of the damage and the removal of the cause. The damage is always visible, but the cause is usually not apparent and very often the client cannot understand why money should be spent on work that seems to him to be unnecessary. For example, the visible defect may be the breaking off of pieces of concrete, thus exposing the steel reinforcement. This is due to the rusting of the steel and its subsequent expansion. The rusting is nearly always due to too little cover. An effective repair can be carried out by simply replacing the concrete that has fallen away, but this procedure would not result in any increased cover to the steel and the trouble would soon recur.

Retaining Walls of Precast Units.

An unusual type of wall made with precast concrete units has been used in the construction of a road near Naples where the gradient transversely to the direction of the road is steep and the ground loose.

Two types of units were used, one forming the wall and the other providing an anchorage (*Fig.* 1). The interstices were filled with loose material to stabilise the wall. The units have at one end a lug that is interlocked with projections on the slabs that form the face of the wall and at the other end they are Y shape. At the face of the wall the transverse Y-shaped units and the longitudinal slabs are secured by steel bars passing vertically through holes in the interlocking lugs (*Fig.* 3).

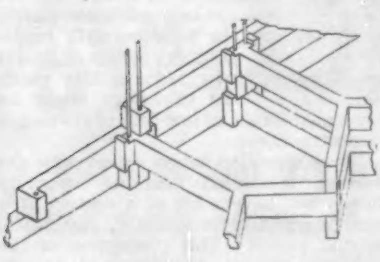

Fig. 1.

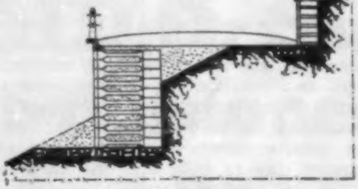

Fig. 2.

The wall was built on a foundation of stones placed in iron baskets, or gabions. For curves in the road of less than 90 ft. radius the walls were built with curved units and the Y-shaped units were adjusted in size to suit. The parapet was also precast and fixed to the overhanging kerb (*Fig.* 2).

The foregoing is abstracted from Technical Digest No. 396, produced by the European Productivity Agency, of an article in "Asfalti Bitumi Catrami", March–April, 1955.

A Flue Built in Refractory Concrete.

REFRACTORY concrete is being increasingly used in the construction of furnaces and kilns and for other purposes where a material is required that will withstand large and rapid fluctuations of temperature. Refractory concrete consists of high-alumina cement and crushed firebrick aggregate. As the work will be subjected to great heat, the reinforcement comprises bars of small diameter or a mesh, but it is advisable not to use reinforcement if plain concrete is suitable. The following notes and the accompanying illustrations relate to the construction in refractory concrete of a flue 60 ft. long by 6 ft. high by 6 ft. wide for Fry's Metal Foundries, Ltd.

Fig. 1.—Shuttering in Position.

The bottom of the trench was covered with refractory concrete about 6 in. thick, and twelve to sixteen hours later wooden shuttering was erected at one end. The top of the shutter was covered with waterproof paper to facilitate its removal. Refractory concrete was then tamped around and over the shutter to a thickness of 6 in. to 12 in. depending upon the imposed load and the amount of insulation required. Because heavy trucks would pass over the flue a layer of steel mesh (*Fig.* 2) was placed in the top slab. When the concrete had set (in a few hours) the shutter was pulled along the trench for casting successive parts of the flue; no joints were provided. Flue junctions and changes of section were formed by shuttering, the angles usually associated with firebrick construction being eliminated with consequent economy. Openings for cleaning and inspection were formed by the shuttering. The concrete comprised 1 part of cement to 6 parts of crushed firebrick with a maximum size of 1¼ in. For concrete about 6 in. thick the proportions recommended are: Coarse firebrick ¾ in. to ¼ in., 2 cu. ft.; fine firebrick ¼ in. down, 2 cu. ft.; high-alumina cement, 1 cu. ft.

Fig. 2.—Concreting in Progress.

In flues where the temperature does not exceed, say, 900 deg. C., crushed clay building bricks or tiles may be used as aggregate. Greater heat insulation may

Fig. 3.—Completed Flue.

be obtained by covering the outer face of the flue with insulating concrete made with high-alumina cement and foamed slag, vermiculite, calcined diatomite, crushed insulating bricks, or other material with good insulating properties cast integrally with the refractory concrete.

Refractory concrete is mixed and placed in the same way as ordinary Portland cement concrete. The aggregates are, however, generally porous and must be soaked with water before use. This is generally done by placing the aggregate in the mixer with a liberal quantity of water and rotating the drum for four or five minutes. Excess water is then drained off and the cement added, followed by more water to produce the required consistency. Curing by sprinkler or wet sacks should be started as soon as the concrete is hard enough to permit this to be done and continued for twenty-four hours.

The aggregate is generally scrap firebricks that are rejected when furnaces are re-lined, and which are crushed to the required size. Such aggregate crushed and graded ready for use is also available, as also is a dry mixture of such aggregate and cement.

"Lift-slab" Method used for Six-story Building.

At the annual convention of the American Concrete Institute held in February last, Mr. James M. Minges described the method of building a hospital, with five stories and a solarium on the roof, now in course of construction at Winsted, Conn. The floor slabs measure 43 ft. by 182 ft. by 8½ in. thick, and the roof of the solarium measures 41 ft. by 80 ft. by 8½ in. thick. The structure is 62 ft. high and is supported by fourteen flanged columns 14-in. wide. The slabs are cantilevered on all four sides during erection. The slabs are prestressed and are designed as two-way flat slabs; the columns are spaced across the building to balance the positive and negative moments.

The prestressing cables are post-tensioned. Each cable comprises ten high-tensile wires of 0·196 in. diameter in a sheath of 1 in. diameter. Most of the cables are continuous for the full width or length of the slabs, and are nearer the top of the slab at the columns and nearer the bottom at midspan. The cables are grouted before the slabs are lifted. When the slabs are hoisted to the required position they are secured in position by pins of 1¾ in. diameter while welding is being carried out; this frees the lifting equipment to proceed with the next slab. It is believed that this method will reduce the lifting time from about four weeks to two weeks.

The columns are erected in two parts. The top slabs for the building are first lifted to the top of the lower part of the columns and temporarily secured. The slabs for the lower floors are then fixed in position, and while the lower slabs are being welded to the columns the top parts of the columns are erected and the upper slabs lifted to position.

Road Improvement in Wales.

It is proposed to start immediately on the improvement of nearly two miles of roadway and bridges on the London–Fishguard road (A.48) at a cost of £394,000. From Lonlas there is to be a roadway to Llansamlet Square; the carriageway will be 30 ft. wide and there will be two footpaths each 10 ft. wide. The three bridges over the Morriston branch railway, the river Tawe, and the Swansea canal will be widened. The work is to be carried out by the Swansea Borough Council. The bridges, which will cost £88,000, have been designed by the Borough Engineer.

New Road and Bridge at Bridgwater.

The Ministry of Transport and Civil Aviation announces that work is to start immediately on the construction of a new road bridge over the river Parrett at Bridgwater, Somerset, as part of a new road, costing £260,000, to ease traffic congestion. The new road, which will be 530 yd. long, will have two carriageways 20 ft. wide, a central space 4 ft. wide, and two footpaths each 15 ft. wide reducing to 8 ft. on the bridge. The bridge will be of reinforced concrete and have a length of 160 ft. in three spans. The design is by Messrs. Mouchel & Partners, Ltd., and the contractors are Sir Lindsay Parkinson & Co., Ltd. The road work will be done by the Somerset County Council.

Protection against Erosion by the Sea.

PROTECTION against the effects of rough seas is difficult because of the unknown forces to be resisted. A rough surface assists in breaking up the waves and provides a more efficient breakwater than a plane surface, and gaps between adjacent blocks are advantageous in permitting relief of the suction exerted by receding waves. This knowledge has been utilised in the design of a precast concrete block known as a "Tetrapod", which was used in the new sea defences in Holland after the great storm of 1953, and which has also been used in the construction of breakwaters in North Africa. The blocks (*Fig.* 1) consist of four intersecting truncated cones and are used in several layers. Those used for the seaward protection of a jetty at Safi, Morocco, weigh 25 tons and have a total height of about 11 ft. Among the advantages claimed for the blocks is that sliding is almost eliminated because of the absence of plane surfaces, that the legs interlock when movement takes place thus rendering further movement less likely, and that porosity of the work is always assured.

The blocks were cast in moulds made of sheet steel 6 mm. thick, formed of three parts that could be bolted together (*Fig.* 2) and four caps closing the ends of the truncated cones. The concrete

Fig. 1.

was a 1 : 1·2 : 5·5 mixture with a water-cement ratio of about 0·5; the density is about 150 lb. per cubic foot, and the crushing strength of 8-in. cubes was about 4250 lb. per square inch at 28 days. Each leg is reinforced by nine longitudinal bars of about ¾-in. diameter and two hoops of ½-in. bars. The lifting hook is a 1¾-in. bar extending about 7 ft. 6 in. into one leg. Consolidation of the concrete

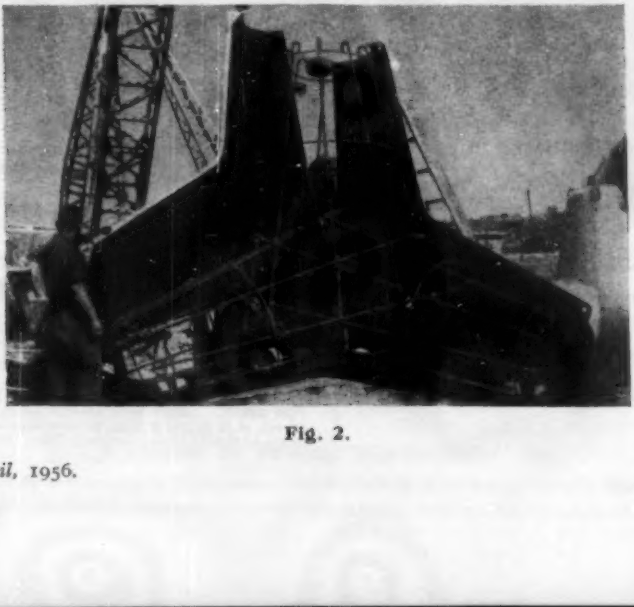

Fig. 2.

was by internal vibrators and the casting of one block required about two hours. The blocks used in Morocco were designed by Ateliers Neyret-Beylier et Piccard-Pictet of Grenoble, France, and are described by M. Y. Bars in a recent number of the French journal "Travaux".

IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY

Department of Civil Engineering

Bursaries in Concrete Technology

NOTICE IS HEREBY GIVEN that the election to Bursaries in Concrete Technology, tenable as from October, 1956, will take place in June, 1956.

Candidates must hold a degree in Engineering at the time of taking up the award, and must also have a good knowledge of the theory of structures.

Bursaries are of the value of £460, out of which the College Tuition Fee of £60 has to be paid; the amount may be increased to a maximum of £760, depending on the qualifications and length and nature of industrial experience. The Bursary is for the normal College session extending from October until the following June.

The course will be postgraduate, and Bursars who successfully complete the course will be eligible for the award of the Diploma of the Imperial College (D.I.C.).

Applications must be received on or before 1st June, 1956, by the DEPUTY REGISTRAR, CITY AND GUILDS COLLEGE, Exhibition Road, London, S.W.7, who will, on written request, send full information and application forms.

The International Association for Bridge and Structural Engineering.

THE fifth congress of the International Association for Bridge and Structural Engineering will be held at Lisbon from June 25 to July 2, 1956. The subjects to be discussed include Loading and strength of bridges and structures, Slabs and curved members in reinforced concrete, Welded steel structures, Structures of steel and light alloys, and Reinforced and prestressed concrete. Full details may be had from the Secretary of the Association, Swiss Federal Institute of Technology, Zürich, Switzerland.

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Published by Concrete Publications Ltd., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
Printed by Butler & Tanner Ltd., The Selwood Printing Works, Frome and London